FLEXURAL STRENGTH OF REINFORCED CONCRETE COLUMNS WITH MIXED ULTRA HIGH AND NORMAL STRENGTH STEEL BARS FOR THE LONGITUDINAL REINFORCEMENT

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ABSTRACT

To give columns a high degree of protection against premature yielding under severe earthquakes, a mixed use of ultra high and normal strength steel bars for the longitudinal reinforcement is being adopted as an alternative. The flexural strength beyond the elastic limit of such kind of columns, however, could not be adequately predicted by use of the code's methods (SNI-1992, NZS3101:1982, ACI318-89). Instead, two methods of Mander et al (1984) and Sheikh and Yeh (1992), mentioned as Method A and Method B in this paper are proposed with a modification in determining the maximum usable compressive strength of concrete. As shown in the experimental results these proposed methods give a satis factory results.

INTRODUCTION

Normally, a column cross section has uniform normal strength of steel for all of their longitudinal bars reinforcement. However, to give columns a high degree of protection against premature yielding under severe earthquakes, a mixed use of ultra high and normal strength steel bars for the longitudinal reinforcement can be used as an alternative [Watanabe et al (1990), Satyarno et al (1993)]. For such kind of columns, the conducted laboratory tests showed that their flexural strength beyond the elastic limit could not be adequately predicted by use of the code's methods (SNI-1992, NZS3101:1982, ACI318-89). Instead, two methods of Mander et al (1984) and Sheikh and Yeh (1992), mentioned as Method A and Method B in this paper, are proposed with a modification in determining the maximum usable compressive strength of concrete.

METHOD A

If a maximum usable strain at the extreme fiber of compressive concrete is prescribed, the compressive force per unit width, Cc, in a column section can be found by assuming an equivalent rectangular stress block, that is:

$$C_c = \alpha f_c' \beta c \qquad 1)$$

where

- α = ratio of compressive concrete stress to control concrete cylinder strength, f_c', in equivalent rectangular concrete stress block,
- β = ratio of equivalent rectangular concrete stress block depth to neutral axis depth,
- c = neutral axis depth.

The compressive force per unit width, Cc, in Eq. 1 acts at a distance $\beta c/2$ from the extreme fiber of the compressed concrete as shown in Fig. 1. Factors α and β in Eq. 1 are found by integrating the area under stress-strain curve using Eq. 3 and from the first moment of area about neutral axis calculated using Eq. 5.

Area under stress-strain curve:

$$\int_0^{\varepsilon_{cm}} f_c \, d\varepsilon = \alpha \beta f_c' \varepsilon_{cm} \qquad 2)$$

therefore:

$$\alpha \beta = \frac{\int_0^{\varepsilon_{cm}} f_c \, d\varepsilon}{f_c' \varepsilon_{cm}} \qquad 3)$$

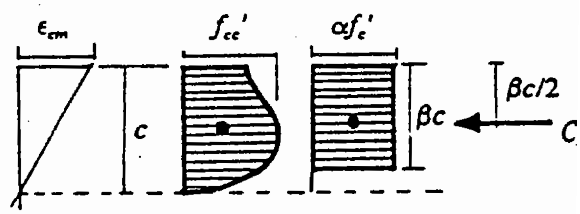

Fig. 1. Equivalent rectangular concrete stress block.

First moment of area about neutral axis:

$$\int_0^{\varepsilon_{cm}} f_c \varepsilon \, d\varepsilon = \left(1 - \frac{\beta}{2}\right) \varepsilon_{cm} \int_0^{\varepsilon_{cm}} f_c \, d\varepsilon \qquad 4)$$

therefore:

$$\beta = 2 - \frac{2 \int_0^{\varepsilon_{cm}} f_c \varepsilon \, d\varepsilon}{\varepsilon_{cm} \int_0^{\varepsilon_{cm}} f_c \, d\varepsilon} \qquad 5)$$

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In Eqs. 2 to 5, the concrete stress, f_c, is determined from a stress-strain model. To avoid difficulties in solving Eqs. 3 and 5, Mander et al (1984) provided graphs to find α and β using their concrete stress-strain model as shown in Figs. 2 and 3.

The main variables in determination of α and β are $\varepsilon_{cm}/\varepsilon_{cc}$ and $K = f_{cc}'/f_{co}'$, where f_{cc}' and f_{co}', are confined and unconfined concrete strength, respectively, and ε_{cc} is strain at peak stress of confined concrete, f_{cc}', see Mander et al (1984) or Satyarno (1993) for detail explanation. For application of Method A the following equation is proposed to calculate ε_{cm} [Satyarno (1993)]:

$$\varepsilon_{cm} = 0.00165\, R_{uhd} + (0.001 + \varepsilon_{cc}) \qquad 6)$$

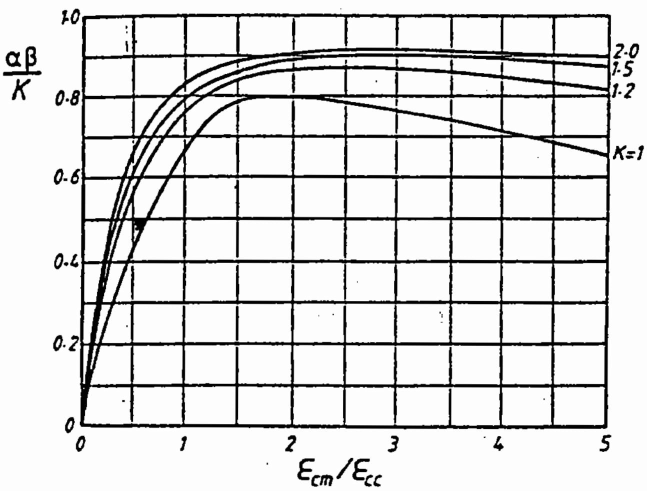


Fig. 2. Graph to find α proposed by Mander et al (1984)

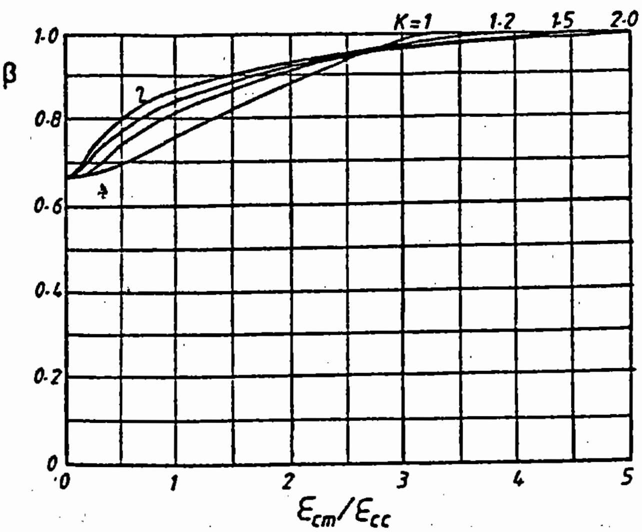


Fig. 3. Graph to find b proposed by Mander et al (1984)

where

ε_{cc} = strain at peak stress of confined concrete in Mander et al (1984) concrete stress-strain model,

R_{uhd} = area ratio of ultra high strength steel to total area of longitudinal reinforcement.

METHOD B

In this method the stress-strain model of concrete proposed by Seikh et al (1992) is used. The model, as shown in Fig. 4. is expressed as follows:

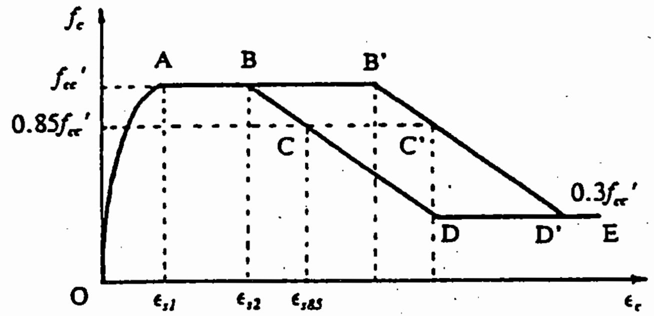


Fig. 4. Stress-strain model of confined concrete proposed by Seikh et al (1992).

for $\varepsilon_c < \varepsilon_{s1}$:

$$f_c = K_s f_{cp} \left[\frac{2\,\varepsilon_c}{\varepsilon_o K_s} - \left(\frac{\varepsilon_c}{\varepsilon_o K_s} \right) \right] \qquad 7)$$

for $\varepsilon_{s2} \geq \varepsilon_c \geq \varepsilon_{s1}$:

$$f_c = f'_{cc} \qquad 8)$$

for $\varepsilon_c > \varepsilon_{s2}$:

$$f_c = K_s f_{cp} \left[1 - Z(\varepsilon_c - \varepsilon_{s2}) \right] \text{ but } \geq 0.3\, K_s f_{cp} \qquad (9)$$

Variables in Eqs. 7 to 9 are found from the following equations:

$$f'_{cc} = K_s f_{cp} \qquad 10)$$

$$f_{cp} = K_p f'_c \qquad 11)$$

$$K_s = 1 + \frac{B^2}{10.58\, P_{occ}} \left[(1 - \frac{nC^2}{5\,.\,5B^2})(1 - \frac{s}{2B})^2 \right] \sqrt{\rho_t}\, f'_s \qquad 12)$$

$$\varepsilon_{sl} = 0.55\, K_s\, f'_c \qquad 13)$$

$$\varepsilon_{sl} = 0.0022\, K_s \qquad 14)$$

$$\frac{\varepsilon_{s2}}{\varepsilon_o} = 1 + \frac{0.81}{C} \left[1 - 5 \left(\frac{s}{B} \right)^2 \right] \frac{\rho_t\, f'_s}{\sqrt{f'_c}} \qquad 15)$$

$$\varepsilon_{85} = 0.225\, \rho_t \sqrt{\frac{B}{s}} + \varepsilon_{s2} \qquad 16)$$

$$Z = \frac{1.0}{1.5\, \rho_t \sqrt{\frac{B}{s}}} \qquad 17)$$

$$P_{occ} = K_p\, f'_c\, (A_{co} - A_s) \qquad 18)$$

where

- ε_{85} = strain at 85 % of f_{cc}' for the descending branch,
- ε_o = strain at peak stress of unconfined concrete,
- ε_{s1} = strain at the beginning of plateau in the concrete stress-strain model,
- ε_{s2} = strain at the end of plateau in the concrete stress-strain model,
- ρ_t = volume ratio of tie steel to the core concrete,
- A_{co} = area of core measured from center to center of the perimeter tie,
- A_s = area of longitudinal steel,
- B = core width measured from center to center of perimeter tie,
- C = distance between laterally support longitudinal bars or 4B/n,
- f_c' = compressive strength of control concrete cylinder,
- f_s' = stress in the lateral steel,
- K_p = ratio of unconfined concrete strength in coulumn to f_c',
- K_s = strength enhancement factor for confined concrete,
- n = number of arches containing concrete that is not effectively confined, also equal to the number of laterally supported longitudinal bars,
- P_{occ} = axial load capacity of unconfined concrete,
- s = tie spacing,
- Z = factor to express the slope of descending branch of monotonic concrete stress-strain model, all dimensions and stresses are in inch. and psi respectively.

They took account of the fact that the behavior of concrete under concentric compression was different from that of the concrete under eccentric compression. The factors that influence the behavior of concrete under eccentric compression are strain gradient in the section and level of axial load. If these factors are taken into account, the previous equations are modified as follows.

To include the effect of strain gradient, Eq. 15 is modified to be:

$$\frac{\varepsilon_{s2}}{\varepsilon_o} = 1 + \left(\frac{0.81}{C}\left[1 - 5\left(\frac{s}{B}\right)^2\right] + 0.25\sqrt{\frac{B}{c}}\right)\frac{\rho_t\, f'_s}{\sqrt{f'_c}} \qquad 19)$$

where c is the neutral axis depth. Eq. 19 shifts the line BD to the line BD' as shown in Fig. 4.

To take the effect of axial load into account in the estimation of f_{cc}', Eq. 10 is modified as:

$$f_{cc} = K_s\, \eta\, f_{cp} \qquad 20)$$

$$\eta = 1.0 - 0.575\,\frac{P - P_b}{f'_c\, A_g} \le 1.0 \qquad 21)$$

where P_b is the balanced axial load based on the code's calculation. Eq. 21 is valid only for the range of $(P-P_b)/(f_c' A_g)$ between 0.1 to 0.5, whereas Eq. 20 indicates that the strength of concrete in the flexural compression zone is reduced as the axial load increases due to a smaller strain gradient in the column section.

In the equivalent rectangular concrete stress block shown in Fig. 5, α and β are found from three parameters which depend on the maximum usable strain at the extreme fiber of compressive concrete, ε_{cm}.

Region 1, for $\varepsilon_{cm} \le \varepsilon_{s1}$:

$$\beta = \frac{4 - \Omega}{2\,(3 - \Omega)} \qquad 22)$$

$$\alpha = \frac{2\,\Omega\,(3 - \Omega)^2}{3\,(4 - \Omega)} \qquad 23)$$

Region 2, $\varepsilon_{s1} < \varepsilon_{cm} \le \varepsilon_{s2}$:

$$\beta = \frac{6\,\Omega^2 - 4\,\Omega + 1}{2\,\Omega\,(3\,\Omega - 1)} \qquad 24)$$

$$\alpha = \frac{2\,(3\,\Omega - 1)^2}{3\,(6\,\Omega^2 - 4\Omega + 1)} \qquad 25)$$

Region 3, for $\varepsilon_{s2} < \varepsilon_{cm} \le \varepsilon_{30}$, where ε_{30} is the strain at $f_c = 0.3\, f_{cc}'$:

$$\alpha\,\beta = 1 - \frac{1}{3\Omega} - 0.075\left(\frac{GD}{D - G}\right)\left(1 - \frac{1}{D}\right)^2 \qquad 26)$$

$$\beta = 2 - \frac{1}{\alpha\beta}\left[1 - \frac{1}{6\Omega^2} - \frac{G\,(2D^3 - 3D^3 + 1)}{20D^2(D-G)}\right] \qquad 27)$$

The value of Ω, D and G are:

$$\Omega = \frac{\varepsilon_c}{\varepsilon_{sl}} \qquad 28)$$

$$D = \frac{\varepsilon_c}{\varepsilon_{s2}} \qquad 29)$$

$$G = \frac{\varepsilon_c}{\varepsilon_{s85}} \qquad 30)$$

For the application of this method, the following equation is proposed to estimate the maximum usable strain at the extreme fiber of the compressive concrete, ε_{cm} [Satyarno (1993)]:

$$\varepsilon_{cm} = 0.00165\, R_{uhd} + (0.001 + \varepsilon_{s2}) \qquad 31)$$

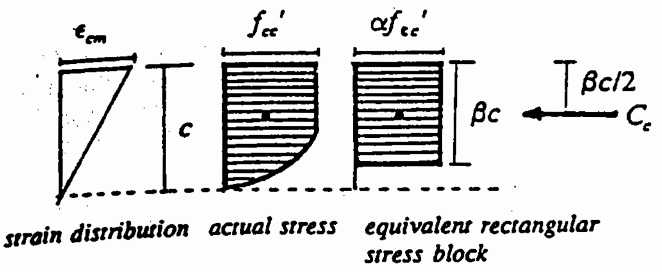


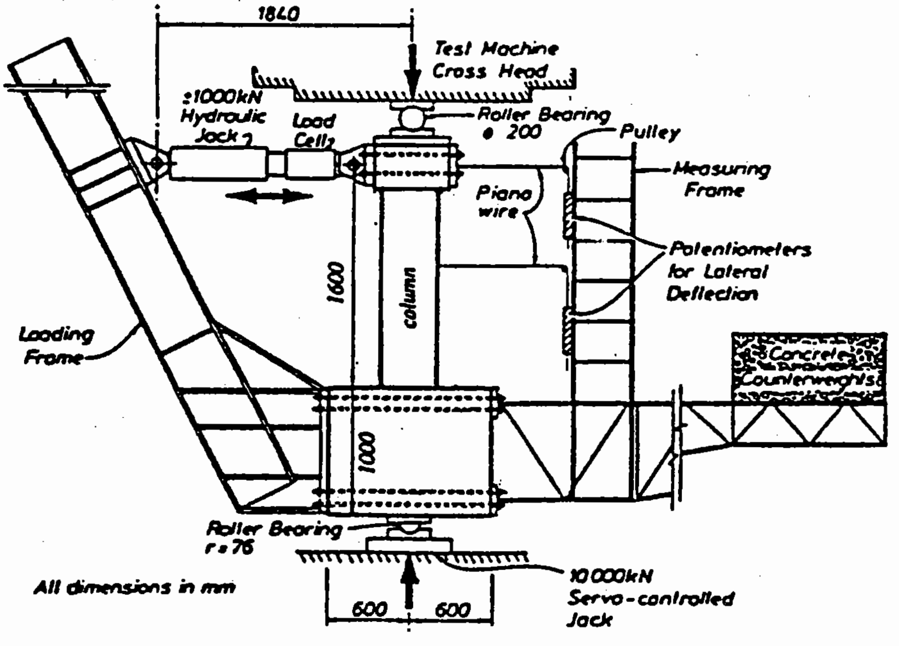
Fig. 5. Equivalent rectangular concrete stress block proposed by Sheikh et al (1992)

EXPERIMENT

Commonly, a reinforced concrete column has only uniform normal strength steel bars for the longitudinal reinforcement. However, in this research ultra high and normal strength steel bars, with specified yield strength of 430 MPa an 1050 MPa, respectively, were used together in a column section. To study the behavior of such column under simulated severe seismic loading, three column units with dimension, arrangement of steel bars and loading set-up as shown in Fig. 6, were tested.

EXPERIMENTAL RESULTS AND DISCUSSION

In this paper, only the flexural strength of the columns is discussed. The cyclic behavior of such column will be presented in the next edition or can be read else where [Satyarno (1993)]. The comparison between flexural strengths of the column units predicted



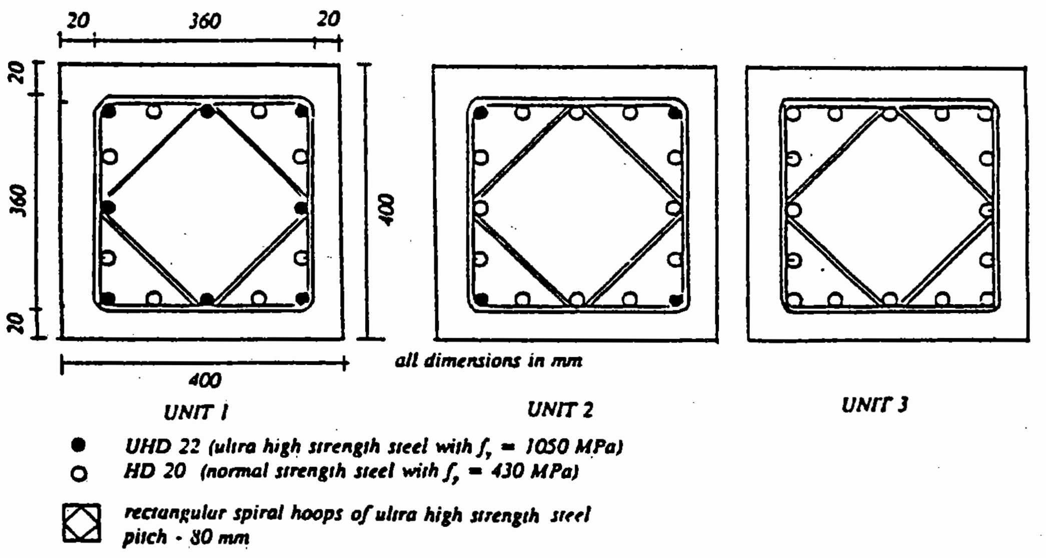


Fig. 6. Configuration of steel bars in column units and loading-set-up.

by the code(s) and the proposed methods can be seen in Table. 1.

Table. 1. Comparison between calculated and experimental results of flexural strength.

Column Units	Calculated flexural strength (kNm)			Experiment (Mexp) (kNm)	Calculated/Experiment		
	Code	Method A	Method B		Code/ Mexp	Met. A/ Mexp	Met. B/ Mexp
1	507	761	718	793	0.64	0.96	0.91
2	500	700	663	710	0.70	0.99	0.93
3	482	592	555	583	0.83	1.02	0.95

Note: - The reduction factor is taken to be 1 in the code calculation.

From the experimental results shown in Tab. 1., it can be seen that:

1. The current code method can not recognize the present of ultra high strength steel in the columns with mixed steel bars for their longitudinal reinforcement.
2. The code calculation results tends to under estimate the flexural strength of columns with mixed steel bars for their longitudinal reinforcement, especially for the ones with more ultra high strength steel content.
3. Method A and Method B give an adequate prediction of the columns flexural strength, but Method A gives a better results.
4. The more the content of ultra high strength steel, the higher the increase of flexural strength as follows:

Table 2. The increase of flexural strength for the increase of ultra high strength steel content.

Column Units	Content of ultra high strength steel (%)	Increase of flexural strength to Unit 3 (%)
1	55.2	36.0
2	29.1	12.17
3	0	-

5. It is proposed that the content of the ultra high strength steel in a column section is calculated using the following equation [Satyarno (1993)]:

$$A_{uhd} = \left[\cdot \left(\frac{0.25}{f_y - 1000} \right) (f_{yuhd} - f_y) + 1 \right] \left(\frac{18A_g}{f_y} - A_s \right) \qquad 32)$$

where

A_g = gross section area of concrete column,
A_s = area of normal strength steel,
A_{uhd} = area of ultra high strength steel,
f_y = yield strength of normal strength steel,
f_{yuhd} = yield strength of ultra high strength steel.

CONCLUSIONS

1. The mixed ultra high and normal strength steel bars for longitudinal reinforcement can give columns a higher of protection against premature yielding under severe earthquake as the flexural strength is improved.
2. The code method to calculate the flexural strength of columns with mixed ultra high and normal strength steel bars for longitudinal reinforcement can not be adequately applied. Instead, the proposed Method A and Method B can be satisfactorily used, in which Method A gives a better results.
3. Although it is not reported in this paper, see Satyarno (1993), columns with mixed ultra high and normal strength steel bars for longitudinal reinforcement also show a good ductility.

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